Lateral Soil Movement Loading on Bridge Foundation Piles

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SUMMARY The effects of a soft layer of soil moving laterally, due to vertical loading from temporary construction fill, are assessed from the aspect of possible damage to piles supporting bridge pier foundations. An analysis using boundary element methods is used to predict possible damage to the bridge piles as an aid in assessing the risk of the piles having failed owing to lateral soil-pile pressures being generated in the soft soil. The analysis results supported the generally held opinion that the piles without a pile cap were very likely to have failed, while indications were such that the piles already capped were not over-stressed.

1. INTRODUCTION

This paper presents the history of piles at the site of a bridge over Moonee Ponds Creek, Melbourne, Australia. The usual procedure for the construction of piled bridge foundations is to place approach embankment fills prior to piling, so that the soft ground settlements and associated lateral soil movements have substantially completed prior to piling works at bridge piers. Even small lateral soil displacements occurring after pile placement may over-load the piles already driven for the piers. The permanent bridge approaches consisted of piled roadways extending back to the existing ground surface to eliminate long term subsoil settlements and lateral movements, which would have undoubtedly affected the bridge foundations, even if low bridge approach embankments had been used. In this case, contractors placed temporary access filling out into the stream course, allowing pile driving equipment to gain access to pier positions. This procedure is less costly than providing a temporary staging structure. An analytical assessment of the potential damage to pier piles as a result of lateral soil displacements caused by the temporary access fills during construction, is also presented.

2. PROBLEM DEFINITION

The plan of the bridge site is shown in Figure 1. Piers 1, 2, 3 and 4 were affected by the lateral soil movements resulting from access fill placement. When the problem was noticed, the pile caps and piers had been built at two of the pier locations and only the piles had been driven at the other two locations. The un-capped piles at piers 1 and 2 were deemed to be beyond repair owing to the excessive measured deflections from the design position. The integrity of the capped piles at piers 3 and 4, which had moved less, remained to be assessed. The elements of the problem are described below.

2.1 Embankment Loading History

The access fill on the Melbourne side of the creek was placed about two months before driving of piles at pier 4 and three months before driving of pier 3 piles. The filling material was a crushed scoria placed over layers of woven and non-woven geotextiles. The maximum depth of scoria (discovered during drilling through the access fills after pile movement problems had become evident) was of the order

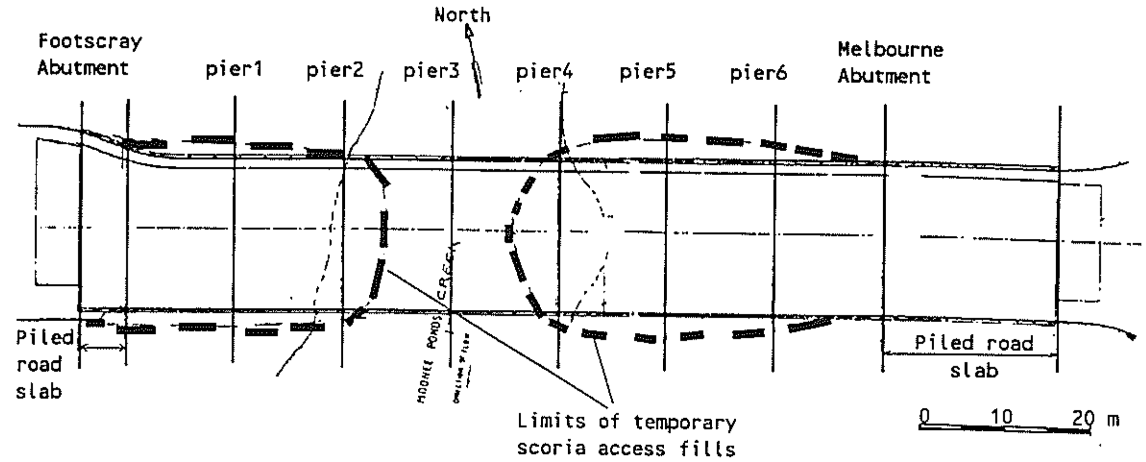


Fig. 1 Bridge site plan - Moonee Ponds Creek, Melbourne.

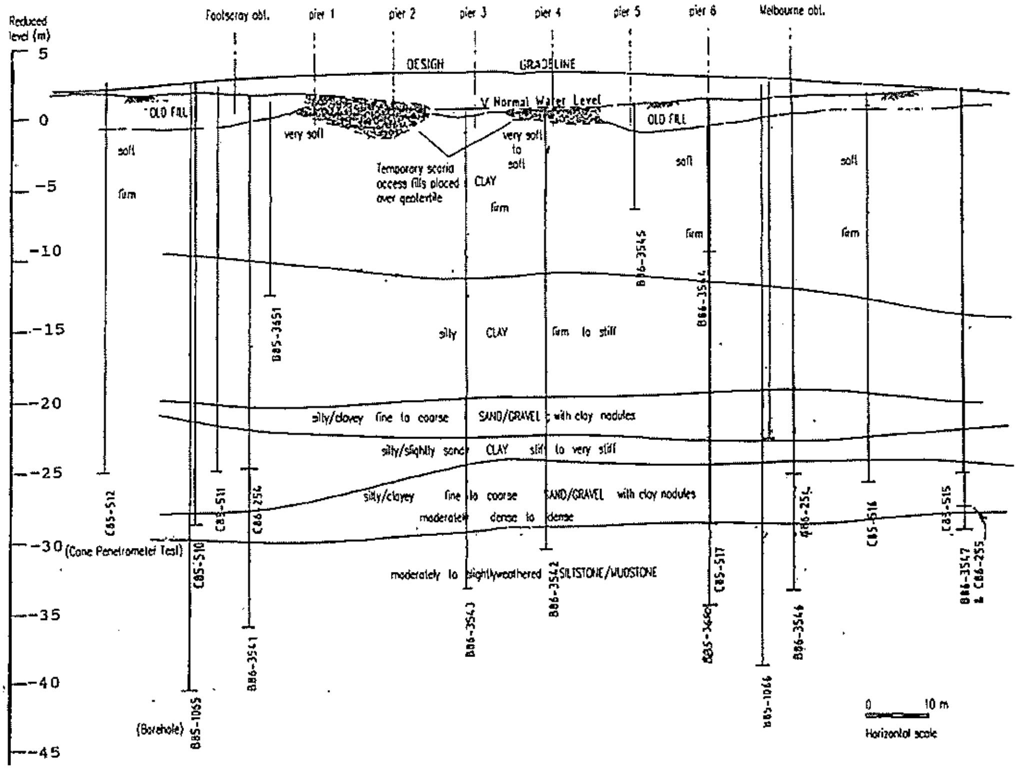


Fig. 2 Sub-surface soil profile - Moonee Ponds Creek bridge site.

of 1 m. Access filling on the Footscray side commenced about four months after that on the Melbourne side. Figure 1 shows the extent of the access fills.

Piles at piers 1 and 2 were driven immediately following access fill placement on the Footscray side, again using a geotextile reinforcing layer under the scoria. After the piling operations were finished the extent and shape of the fill were altered to try and limit the lateral soil movements. The access filling was found to be up to 3 m thick on the Footscray side of the creek.

Construction personnel were unaware of the exact volumes of scoria being placed for the access fills, although it was clear that additional material had been placed from time to time to raise the working platform above the water level. Apart from the obvious vertical settlement, the consequences of the scoria placement with respect to the effects on the piles were not fully appreciated. Following the observations of excessive pier and pile movement, subsequent investigations showed that considerable subsoil flow and displacement had occurred, presumably during placement of the fill material.

2.2 Soil Conditions

The creek bed consists of a very soft wet clay with mud flats exposed at high tide. The subsoil profile indicates very soft to firm estuarine clay to depths of up to 15 m, see Figure 2. Over the uppermost 5 m of this clay, the undrained shear strength varies from 5 to 10 kPa. This layer is weakest on the Footscray side, probably associated with the failure of a temporary site investigation access fill, placed without geotextile strengthening, about two years prior to bridge construction.

Beneath the estuarine clay are interbedded firm to stiff clays, dense sands and gravels overlying the siltstone bedrock at a depth of about 30 m. Figure 2 indicates the arrangement of the major soil layers.

2.3 Foundation Description

Reinforced concrete piles of 350 mm square section, driven about 30 m to found on siltstone, were used in rows of 10 to support the bridge piers. Six spans of 12.8 m supported the bridge and typical axial loads of 187 kN were used in design. The bending moment to cause steel yield was taken to be 130 kNm with no axial load. The concrete piers were typically 500 mm wide, 2 m high and 19 m in length with a pile centre to centre spacing of 1.96 m.

3. ANALYSIS OF PILE RESPONSE

The response of the soil was modelled as an elastic continuum, as employed by Poulos (1973), and based upon the solution of Mindlin (1946), as developed by Douglas and Davis (1964). The pile model is based upon a finite difference formulation of the fourth order differential beam equation. Combination of these two models is described in detail by Hull et al (1991) and is implemented in the computer program PALLAS (Piles And Lateral Loading AnalysiS).

Within the analysis the effect of the soil movement on the pile is accounted for, but the source generating the soil movement has no other influence. The value of the distributed load generated at the interface controls the non-linear response of the model, by the appropriate condition of pile-soil displacement compatibility or zero incremental load being imposed at the pile and soil interface.

The problem had to be simplified such that the major influences were incorporated while the analysis itself remained tractable. Firstly, the plane of the axis of the bridge was chosen as the plane in which the modelling was undertaken. Measurements of pile movements had a symmetry about the bridge axis that suggests errors from this approximation may not be severe. Secondly, the precise soil loading history and subsoil movements were not known and could only be deduced from the measured pile movements, the soil layering and strength data, and information from the inclinometer casing installed through the Footscray fill three months after placement. Representative soil profiles and loading histories were then developed in order to assess the magnitude of the likely problem, rather than attempting a precise reconstruction of the site and history. Thirdly, the influence of the group of piles was omitted, as preliminary analyses including group effects showed no significant alteration from a single pile response.

4. MODEL PARAMETERS

To enable prediction of the likely distribution of bending moment at any point in time a number of model parameters must be estimated, as well as the soil movement profile history.

For analyses of problems with large lateral soil movement, the value that is used for the limiting soil "pressure", p_u transmitted to the pile exerts a major influence on the results. This influence arises because satisfying of the equations of equilibrium for the pile actually determines the pile bending moment and shear force distributions in the regions of failed soil. The pile displacements and rotations are then determined by the stiffness response of the remaining linear soil regions. The important parameters can be seen to be the soil stiffness and the limiting "pressure" developed in the soil due to relative lateral movement of the pile. The soil stiffness is controlled by the value of Young's modulus E_s used and, like the value of p_u, is often correlated with undrained soil shear strength c_u.

4.1 Program Parameter Selection

The values of program input parameters were based upon the distribution of soil undrained shear strength inferred from both Swedish Fall-Cone (Hansbo, 1957) test results and triaxial tests on samples bored from the site, cone resistance readings (assuming $q_c/15 < c_u < q_c/25$) and previous experience at nearby sites (Poulos and Hull, 1989). It was considered appropriate to assess three possible distributions of shear strength; namely a value of 20 kPa constant with depth, a value proportional with depth ($c_u = 3z$ kPa, where z is the depth in metres) and a case intermediate to these having a value of 5 kPa at the soil surface and 45 kPa at 15 m depth. These distributions have been designated by the letters "a","b"and "c"in the figures and are plotted in Figure 3 as functions of depth.

To account for the proximity of the soil surface the chosen failure "pressure" at the pile-soil interface incorporated a bi-linear variation of the coefficient relating p_u to c_u, starting at 2 at the surface and increasing proportionally with depth to 9 at a depth of four diameters and deeper. The distributions of p_u are also plotted in Figure 3.

Correlations of the soil Young's modulus with shear strength suggest c_u multipliers of between 200 and 1000 (Poulos and Hull 1989) and a multiplier of 500 was chosen as the most appropriate. Poisson's ratio for the soil was taken to be 0.45, but this has little effect on the results of the analysis.

A likely distribution of soil movement was more difficult to assess, as it was felt that a reversal of soil movement may have occurred because of the influence of later fill placement closer to the piles and opposite the previous fill. Readings were taken of an inclinometer between piers 1 and 2, over a period of 3 weeks in January 1989 (10 months after filling commenced, and during which no more fill was placed). The readings showed that a movement of about 7 mm occurred at the soil surface, linearly decaying to zero at a depth of 10 m. Groundline pile movements in piers 1

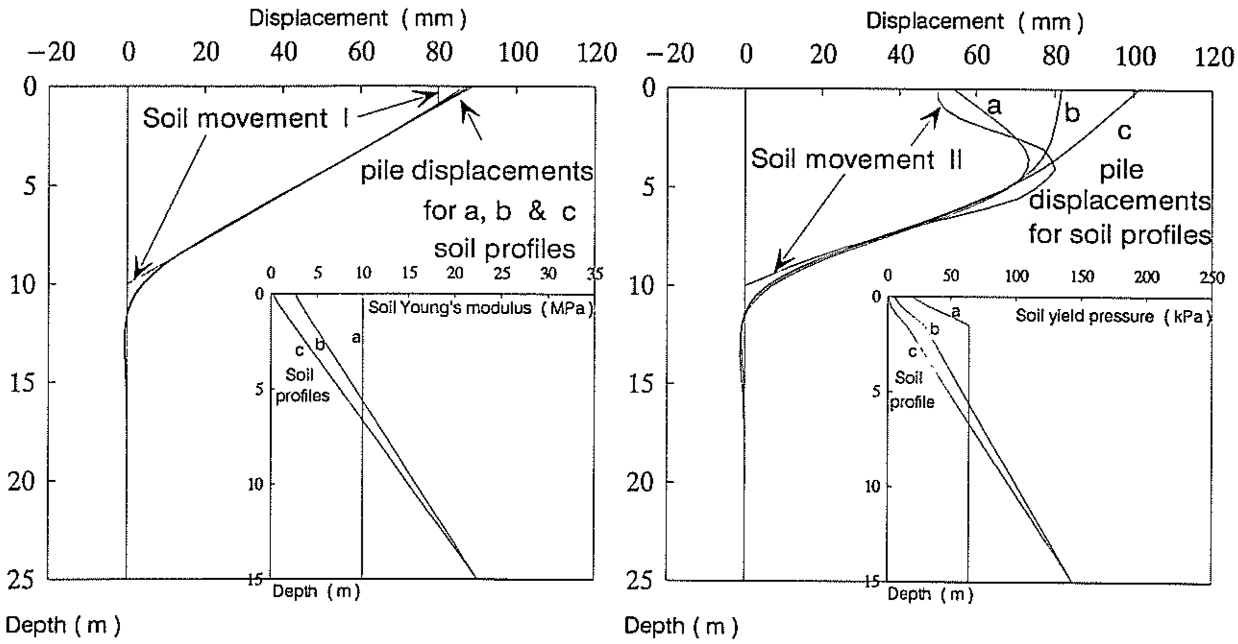


Fig. 3. Deflected shapes of piles for two soil movement profiles and three soil profiles.

and 2 were about 400 mm during the 2 months prior to installation of the inclinometer. The softer soil at the Footscray side, where piers 1 and 2 were located, and the fact that the piles at these locations were driven immediately after placement of fill on the Footscray side explains the excessive deflections they suffered. Piles in piers 3 and 4 would be expected to have experienced less lateral soil movement since; the soil was not as soft, the piles were driven over 2 months after placement of the Melbourne fill and the position of pier 3 was mid-way between the two areas of fill, where some null displacement point might be expected.

With due regard to the above, two distributions of soil movement (I and II) were adopted for the analysis of the piles, which were considered to be upper limits to the expected movements at any pier, as shown in Figure 3.

Other parameters were a pile bending stiffness, EI of 42,600 kNm^2, pile diameter of 355 mm and a modelled pile length of 20 m (the lower 10 m of the pile was in-effective, the critical length being about 6 m).

5. RESULTS

Figure 3 presents the deflected shapes of a pile in the pier for the two adopted soil movement profiles and the three undrained soil shear strength distributions. Only the second soil movement profile, which has a double curvature, causes the pile deflected shape to depart from the imposed soil movement profile. The degree of departure is then related to the properties of the soil in the upper 5 m; the stiffer and stronger soil for the first case causing the pile to more closely follow the imposed soil movement.

Both adopted soil movement profiles have a positive maximum bending moment at a depth of 10 m, where the soil movement is zero, as seen in Figure 4. But, the second soil movement profile results in larger bending moments and a wider variation with soil strength and stiffness, especially at about 4.5 m depth where a negative maximum bending moment is developed. It should be noted that all the cases produce bending moments close to, or exceeding, that required to yield the pile.

The behaviour of the real problem is most unlikely to be the result of a purely monotonic process and, to investigate a possible result of non-monotonic loading, a history of pile loading was simulated. Although employing a simpler loading than that which would have actually occurred, the results illustrate several important aspects of pile response to lateral soil movement. In Figure 5 the response of the pile to a varying imposition of the second soil movement profile (II) and using the soil profile "b" is presented as a function of time, with the major events indicated by symbols in the figure and explained in the legend.

The upper plot in Figure 5 defines the sense of the pile response and presents the movement occurring at the soil surface and the movement of the pile at the groundline as functions of time in months. The pile is seen to move more than the surface soil, and subsequently, upon reversal of the soil movements, to reach the initial pile position some time after the surface of the soil had returned to its initial position. This is a result of permanent deformations in the soil, but there is no indication from the pile deflections that the soil-pile system has reached a steady state of failure.

The values of the maximum positive and maximum negative bending moment plotted in the centre of Figure 5 show that the two maximums are essentially of equal magnitude. But also, that there is a period during the month of November in which small moments are experienced, just before the soil movements changes sense. Both curves of the maximum moments demonstrate that the pile has exceeded the yield moment, both before and after the soil movements reverse direction.

In the lower plot of Figure 5 the pile rotation at the groundline is presented. During the month of September the rotation is seen to reduce (even though the soil movement is still increasing) and is seen to be zero in mid-October

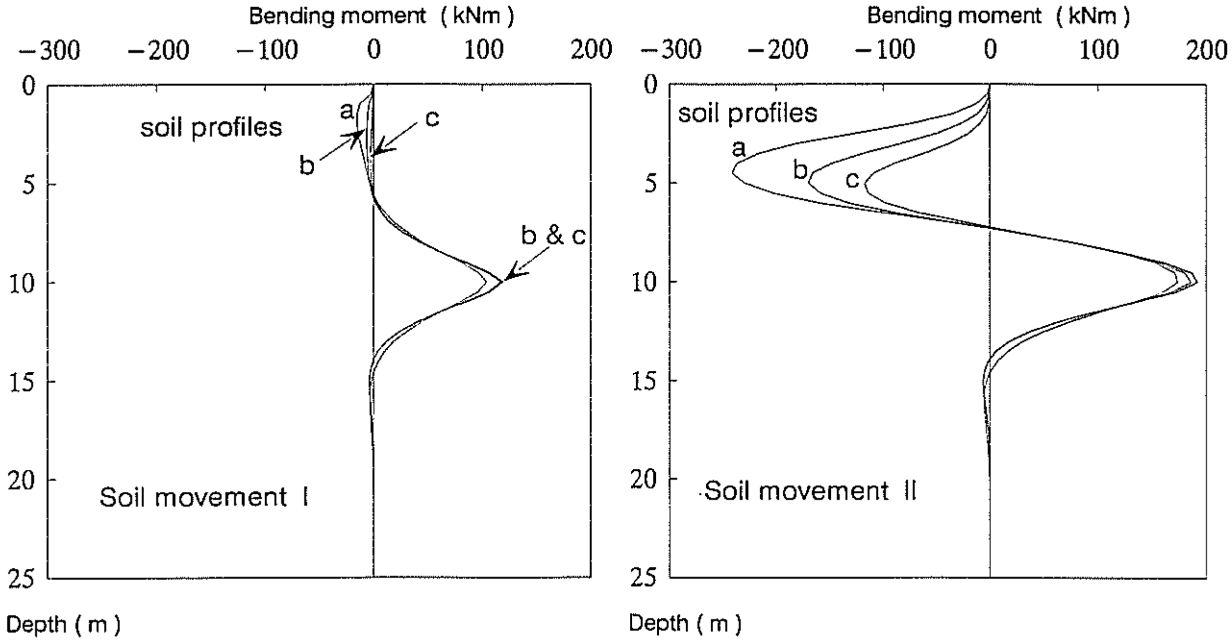
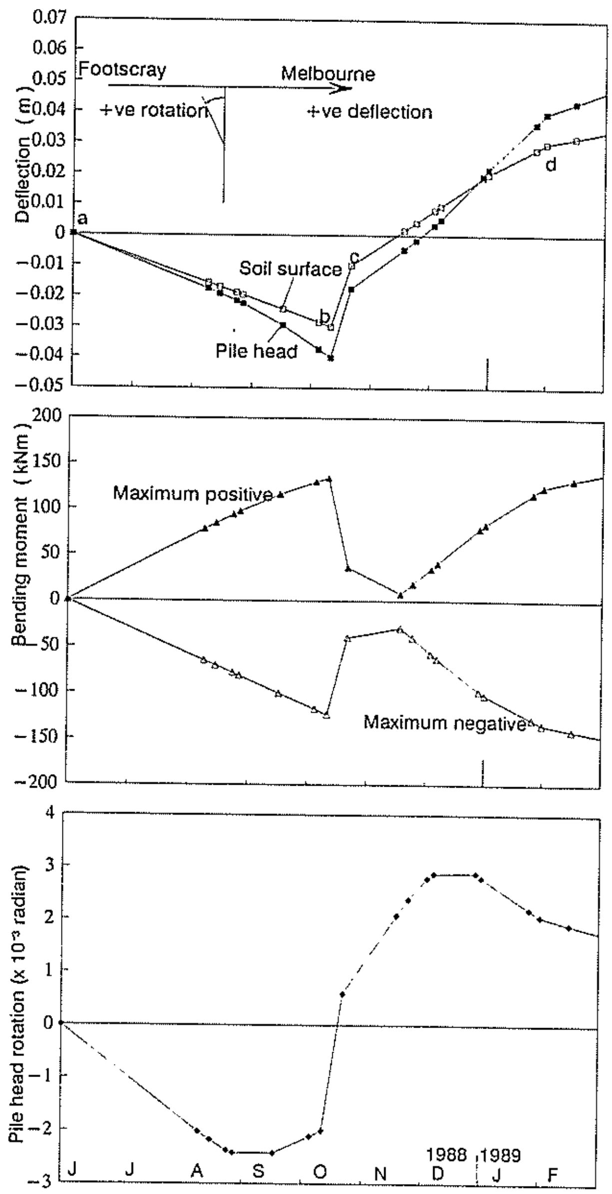


Fig. 4 Bending moment distributions for two soil movement profiles and three soil profiles.



a Beginning modelling of Melbourne fill influence
b Commencement of Footscray fill
c End of main filling operation, Footscray fill
d Removal of some of the Melbourne fill

Fig. 5 Approximate modelling of the pile loading history.

(even though the soil and pile movements are non-zero). Such a reduction of rotation again occurs after December. The progressive growth down the pile of the yielded zone of soil is responsible for these unusual facets of the response.

6. CONCLUSIONS

The most critical aspect of the modelling concerned the distribution of subsoil lateral movements that had occurred after installation of the piles. Since these lateral movements were not accurately known, alternative possible distributions had to be developed taking account of the measured pile movements, subsequent inclinometer readings and soil profile data.

The analyses demonstrated that the deflections of the piles followed the subsoil movements closely, since the pile stiffness was insufficient to offer any significant restraint to the soil movement, except when the lateral soil movement distribution varied rapidly with depth, e.g. near the soil surface for the second soil movement profile. Confirmation was provided that the piles at piers 1 and 2 had probably failed in bending. The piles at piers 3 and 4 appear to be overstressed when the proposed severe soil movements are applied. However, it has been argued that the soil movement would be less severe at piers 3 and 4, and this will lead to reduced bending moments.

The computer modelling provided a valuable insight into the response of the piles to subsoil movements and greatly assisted bridge project decision making with respect to remedial actions. It was decided to replace the piles at piers 1 and 2 and to jack piers 3 and 4 back to their designed positions. Although the lateral pier deflections at piers 3 and 4 were no more than 75 mm, the eccentricity of vertical loading would have been sufficient to overstress the piles in service.

The experience with pile and pier movement at this site emphasised the need for careful consideration of the risk of the effects of temporary construction works over soft, compressible soil. The construction personnel were not aware of a developing problem or the risk associated with the unusually large amounts of scoria being used to construct the access fills.

7. ACKNOWLEDGEMENTS

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